INFLUENCE OF HF2V DAMPING DEVICES ON THE PERFORMANCE OF THE SAC3 BUILDING SUBJECTED TO THE SAC GROUND MOTION SUITES

G. W. Rodgers[1], J. G. Chase[2], G. A. MacRae[3], T. Bacht[4], R. P. Dhakal[5], and J. Desombre[6]

ABSTRACT

Recent advances in energy dissipation for structural systems can create structural connections that undergo zero sacrificial energy absorbing damage, even at extreme story drifts. However, questions exist around the ability of such structures to re-center after a major event. In this paper, the seismic performance of the as-designed SAC LA3 seismic frame with rigid moment connections at the beam ends is compared with the same frame using semi-rigid connections with high force-to-volume (HF2V) lead dissipators. Non-linear dynamic analysis is preformed using Abaqus™. With respect to re-centering, the presence of the gravity frames in the model is also considered. It was found that the placement of dissipators, ignoring the effect of gravity frames, caused a 12% increase in period due to the decreased stiffness of the connections. During design level ground shaking the semi-rigid connections with HF2V dissipators have slightly lower accelerations, up to an 80% increase in peak drift, and a 200% increase in the permanent displacement compared to the as-designed case, but no structural damage is expected. When gravity frames are considered, the floor accelerations decrease further, the peak displacements do not significantly change, but the residual storey drift ratios reduce to approximately 0.17%. This result is less than one half that of the as-designed frame, where typically gravity frame effects are not considered. The addition of braces with a stiffness 20% of the pushover stiffness ensures that the structures can re-center after any given event to within construction error. The realistic non-linear dynamic analyses combining HF2V dissipators with gravity frames and well-designed non-structural elements creates a system with almost no structural damage and low residual displacements.

Introduction

Steel moment frame structures exposed to moderate or strong ground motions are designed to accept damage in the beam end plastic hinge zones or in the beam-column joint panel zones to dissipate seismic energy. Large permanent displacements may be present at the end of the earthquake shaking. Repair costs for such damage, and the consequent downtime, can be substantial creating significant economic and business impact.

[1] Postdoctoral Researcher, Dept. of Mechanical Engineering, University of Canterbury, Christchurch, New Zealand
[2] Professor, Dept. of Mechanical Engineering, University of Canterbury, Christchurch 8140, New Zealand
[3] Associate Professor, Dept. of Civil Engineering, University of Canterbury, Christchurch 8140, New Zealand
[4] Visiting Researcher, Dept. of Mechanical Engineering, Univ. of Canterbury, Christchurch 8140, New Zealand
[5] Associate Professor, Dept. of Civil Engineering, University of Canterbury, Christchurch 8140, New Zealand
[6] Visiting Researcher, Dept. of Mechanical Engineering, Univ. of Canterbury, Christchurch 8140, New Zealand

The use of a Damage Avoidance Design (DAD) approach, with relatively damage-free connections, can reduce repairs and minimize disruption, which would substantially reduce economic impacts. Damage avoidance may be achieved in moment frames by using specialised devices whose dissipative performance does not degrade on subsequent cycles of use. Ideally, such devices would perform in a consistent and repeatable manner on every response cycle throughout the life of a structure, and would not require maintenance or replacement after a seismic event.

Energy dissipation devices using lead were proposed by Robinson and Greenbank (1976) to absorb energy in a controlled, repeatable manner as a base isolation system. Lead is ideal for this purpose due to its unique rheological properties, low re-crystallisation temperature, and ability to allow any residual compression forces in the device to creep back towards zero over time (Rodgers et al. 2007; Rodgers et al. 2008b). A summary of the state of the practice developed from this early work is given by Cousins and Porritt (1993). While these devices were ideal for their intended purpose (Robinson 1995), they were too large to fit within standard structural connections, despite having the necessary resisting force and energy dissipation capacity.

Recently developed, compact high force-to-volume (HF2V) dissipation devices (Rodgers et al. 2007) can fit directly into structural connections (Mander et al. 2009; Rodgers et al. 2008a; Rodgers et al. 2007; Rodgers et al. 2008b) to enable damage avoidance connections. The device consists of a cylinder filled with lead through which a bulged shaft passes. Shaft motion forces lead around the bulge, providing a resisting force. These devices may be modeled as weakly velocity sensitive non-linear viscous dampers, where higher velocities yield a greater force. Relatively inexpensive to manufacture, they have been experimentally characterized with details published elsewhere (Rodgers et al. 2007; Rodgers et al. 2008b). Energy is dissipated on every response cycle without strength degradation or damage (Rodgers et al. 2008c), in contrast to conventional steel connections or connections with sacrificial dissipators (Bradley et al. 2008; Li 2006).

To address steel moment frames, it is necessary to answer the following questions:

1) Can HF2V devices be realistically used in steel frames?
2) Can these devices in steel joints be modelled appropriately?
3) Are there ways to minimize permanent displacements of the structure?
4) How does the response change when using these devices, instead of conventional connections, in modern steel moment resisting frames?

This paper conducts several non-linear analyses to compare the seismic performance of the SAC Steel Project 3-story structure with and without the HF2V devices.

Applying HF2V Devices to Steel Frame Structures

Experimental results of the HF2V joint showed damage-free performance for interstorey drifts of up to 4% (Mander et al. 2009). Similar results were obtained in DAD concrete connections (Rodgers et al. 2008a; Solberg 2007). A simple model with low computational demand is required to analyze these devices in large non-linear dynamic simulations. Therefore, a rotational hinge element, containing elastic, plastic and velocity dependent (damping) aspects, was developed to match experimental results. It was implemented in ABAQUS. A comparison between the experimental data and this model developed is shown in Figure 1. The moderate differences at large story drifts are attributed to experimental setup and experimental control issues (Mander et al. 2009) and not to specific device behavior. The model is thus sufficiently reliable for large structural system analyses.

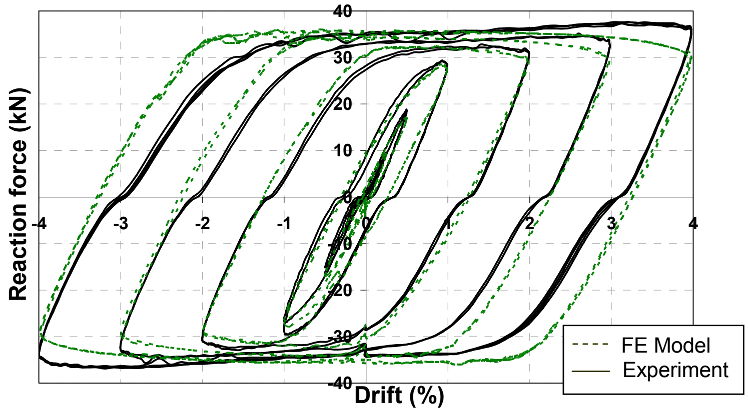

Figure 1: Finite element and experimental results for HF2V steel connection.

The HF2V devices exhibit behaviour similar to that from an elastic perfectly plastic (EPP) structure as shown in Figure 1. Figure 1 was created by combining independent finite element models of a test connection and HF2V device developed to match experimental results. It was implemented in ABAQUS. Placed in a real structure, P-delta effects on the frame are likely to cause the curve to have a negative post-elastic stiffness ratio, *r*, where *r* is shown schematically in Figure 2a. Single-degree-of-freedom (SDOF) oscillators with $r \leq 0$ have little inherent dynamic re-centering capability, as shown by the average residual displacement ratio, $\Delta_r/\Delta_{r,max}$, in Figure 2b, where Δ_r is the residual displacement and $\Delta_{r,max}$ the maximum possible residual displacement based on fracture limits (MacRae and Kawashima 1997). Because of the hysteresis curve shape, even if no damage occurs, the structure may have large permanent displacements making it vulnerable to aftershocks and difficult to straighten after an earthquake.

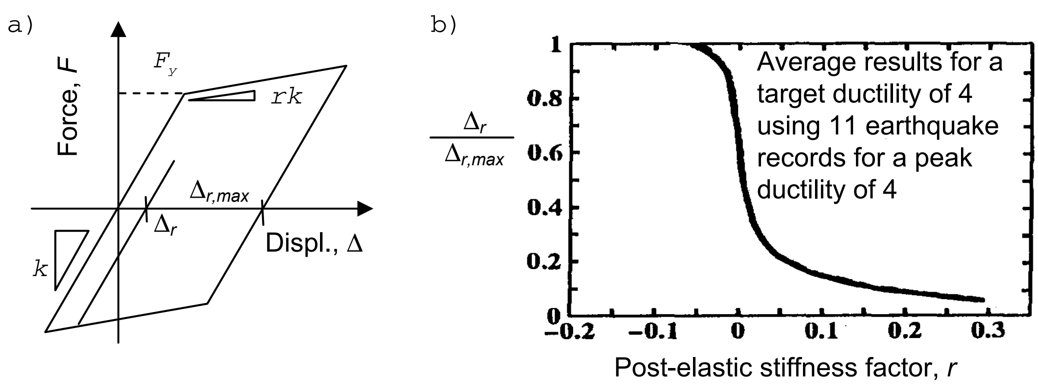

Figure 2: Effect of Hysteresis Curve on Residual Displacement Ratio: a) Schematic Hysteresis curve, and b) Average Residual Displacement Ratio (Kawashima et al. 1998).

Figure 2 shows that one means of improving response is to increase the post-elastic stiffness factor, *r*, until it is significantly positive. Another method in multi-storey frames is to consider the effect of continuous columns over the height of the structure. Such columns, which include the seismic in-plane columns, the seismic out-of-plane columns and all gravity columns, provide continuity between stories. Thus, the response of the overall frame is not like that of the SDOF oscillators in Figure 2. In fact, MacRae et al. (2004) have shown that continuous columns reduce the drift concentrations in individual stories. In many dynamic analyses, the effect of column continuity is often not considered, except in the seismic frame analyzed, and gravity frame effects are thus often ignored. Based on these results, large permanent displacements may be mitigated by either increasing the post-elastic stiffness of each story, or by providing/considering column continuity.

While post-elastic stiffness of the HF2V device is close to zero, the post-elastic stiffness of the total storey in a steel frame may be greater than zero, primarily as a result of the rotational stiffness of the gravity beam end connections. This added stiffness contribution, which can be controlled to some extent in the design and is in addition to the contribution of column continuity, is investigated in this research.

Frames Analyzed

The structural system used in this investigation was developed as part of the SAC Steel Project (Krawinkler and Gupta 1998; SAC 1999). The structure and earthquake suite used in this research were developed for the Los Angeles area (Sommerville et al. 1997). The specific structure in this study is the three-storey steel building designed for Los Angeles, also called SAC-3 or SAC LA3, with moment resisting frames only at the periphery (Krawinkler and Gupta 1998). Each bay has centerline dimensions of 9.14m x 9.14m and the columns extend over the 3 stories of 3.96m height. The structure is nearly uniform in both orthogonal directions. The horizontal seismic weight per frame at levels 3, 2 and 1 are 5200kN, 4800kN and 4800kN.

For the 2-D analyses here, only the east half of the building is modeled. The seismic frame is modeled directly, but to further reduce the total number of degrees of freedom, the other columns on the east half of the building are merged into a single "consolidated gravity column" (Axis E) by summing the stiffness and strength of the individual columns considering deformation in the N-S direction. The consolidated gravity column is slaved in the horizontal direction at every floor to the seismic frame to form a complete two-dimensional model shown in Figure 3. Pins at the ends of the beams in the right hand bay represent perfectly pinned connections. With these pinned connections, the bay width in the right hand bay is unimportant. However, this width was assumed to be one half of the actual bay width for convenience in different case studies, as discussed later.

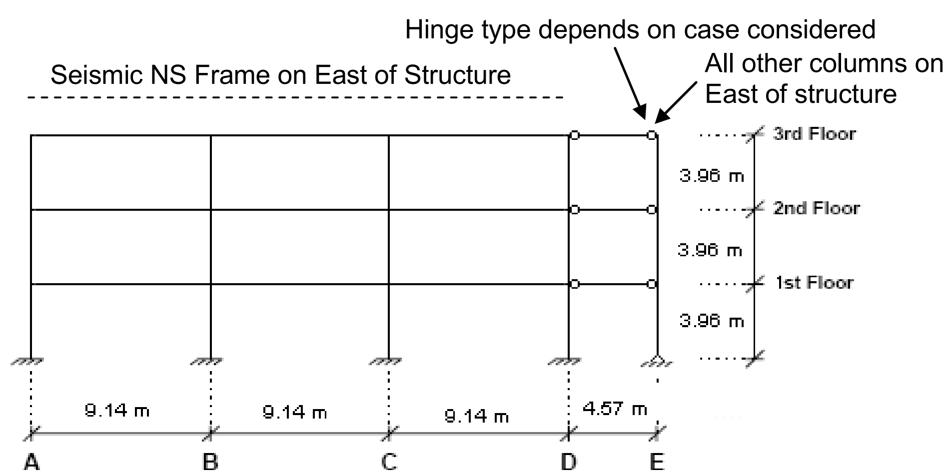

Figure 3: Model 1 - As-Designed SAC3 structure.

The second and third models considered, Model 2 and Model 3, used HF2V devices in the seismic frame, rather than the as-designed rigid beam-column connections. These modified connections are shown as rectangular boxes between grid lines A – D of Figure 4. As a result, it is expected that this change in connection design will affect the structural stiffness and natural period, as well as its ability to dissipate energy.

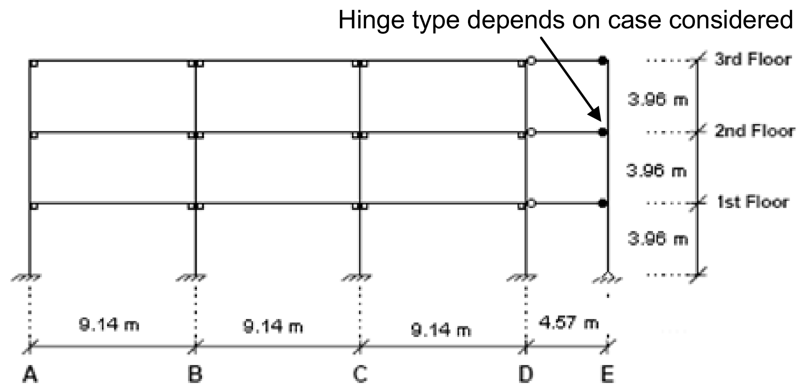

Figure 4: Models 2 and 3 - With HF2V devices in the seismic Frame.

The HF2V device force capacity is chosen to achieve the base shear strength required by FEMA 450 (2003). The seismic base shear was determined by Equation (1) where C_s is the seismic coefficient and W is the structure's seismic weight.

$$V = C_S W \tag{1}$$

where C_s was determined by Equation (2), S_{DS} = the design spectral response acceleration parameter, R = the response modification factor and I = the occupancy importance factor.

$$C_S = S_{DS} / (R / I) \tag{2}$$

For the Los Angeles area, soil type D, an R factor of 8, and a period, T, of 1.0s, means $C_s = 0.05$. Hence, the design strength for this structure is 5% of its seismic weight. Model 2 was provided with devices that allowed this base shear value to be reached.

Due to the possibility of overstrength, and to have a more reasonable comparison with Model 1, Model 3 was designed to resist twice this base shear, or 10% of the total weight. In both Models 2 and 3, HF2V device capacities were thus selected so that shear resistances at each level over the seismic frame height were proportional to the weight above.

For each of the seismic frame models, Models 1-3, three separate cases for the gravity beam end connection rotational stiffness and strength were considered. Parameters describing the bilinear beam end rotation stiffness and strength values were provided beside Column E. These parameters are given in Table 1.

- **Case 1** represents the case of a perfect pin with no rotational stiffness ($k = 0$).
- **Case 2** describes much less effective connections with $k = 1.7EI/L$ and M_s of 50% of M_p.
- **Case 3** describes a major effect of the gravity beam (and slab) end connection, where the rotational stiffness, $k = 2.5EI/L$ and the strength, M_s, is 100% of the plastic moment of the beam, M_p.

The values chosen in Cases 1 and 2 are based on previously published experimental studies (Liu and Astaneh-Asl 2000; Liu and Astaneh-Asl 2004). In the span D-E beam, the point of inflection in the gravity beams is 4.57m from Column E, which is the expected location of the point of inflection due to seismic forces. This point is shown by the white circles representing perfect pins on the right of column D in Figures 3 and 4.

Table 1: Analysis matrix and model/device properties for each case.

Model	Model Characteristics	Case	Gravity Beam End Connection for Each Case
1	As- designed seismic frame	1	$k = 0$
		2	$k = 1.7EI/L$, $M_s = 50\%M_p$
		3	$k = 2.5EI/L$, $M_s = 100\%M_p$
2	With HF2V Devices - Design Base Shear is 5% of Weight	1	$k = 0$
		2	$k = 1.7EI/L$, $M_s = 50\%M_p$
		3	$k = 2.5EI/L$, $M_s = 100\%M_p$
3	With HF2V Devices - Design Base Shear is 5% of Weight	1	$k = 0$
		2	$k = 1.7EI/L$, $M_s = 50\%M_p$
		3	$k = 2.5EI/L$, $M_s = 100\%M_p$

The different models (representing the seismic frame characteristics) and the different cases (representing the gravity beam end connection bilinear parameters) are given in Table 1. Model 1 Case 1 (M1C1) represents the SAC LA3 building design as it is generally analyzed with no extra consideration for gravity frame effects. The fundamental periods varied from 1.0s for M1C1 to 1.12s for M3C1. Initial stiffness proportional Rayleigh damping of 5% was used in the first mode. Velocity dependence of the device dissipation was also incorporated, as reported by Rodgers et al. (2007; 2008b).

Dynamic inelastic time history analyses were conducted with ABAQUS using the LA medium suite of the earthquake records from the SAC Steel Project (Sommerville et al. 1997). These 20 earthquake records have a magnitude between 6.5 - 7.25, epicentral distances of 5 – 40 kilometers and an exceedance probability of 10% in 50 years. They are thus design basis earthquakes (DBE).

Results and Discussion

The total floor accelerations, relative residual roof drifts and relative storey drifts are listed in Tables 2 to 4. In each case, the median (50th percentile) and appropriate lognormal standard deviation (Limpert et al. 2001) are calculated based on the 20 data points, one from each ground motion response in the earthquake suite used.

Table 2: Response for Gravity Beam End Connection Case 1 (k=0).

	Model 1		Model 2			Model 3		
	As built		5 % Baseshear			10 % Baseshear		
	lognormal mean	deviation	lognormal mean	deviation	change [%]	lognormal mean	deviation	change [%]
Peak Acc Floor 1	8.062	1.640	7.580	1.597	-6.362	7.423	1.618	-8.613
Peak Acc Floor 2	6.892	1.535	5.437	1.668	-26.767	5.412	1.665	-27.341
Peak Acc Floor 3	8.250	1.483	5.772	1.534	-42.938	5.626	1.535	-46.636
Relative Roof Drift	0.185	1.916	0.359	1.430	94.295	0.327	1.413	76.582
Residual Roof Drift	0.039	2.657	0.120	2.366	207.088	0.067	5.787	70.848
Interstorey Drift 3-2	0.075	1.353	0.133	1.380	77.848	0.122	1.357	63.188
Interstorey Drift 2-1	0.070	1.383	0.122	1.435	73.323	0.111	1.411	58.646
Interstorey Drift 1-0	0.068	1.357	0.110	1.444	60.480	0.089	1.684	30.430

Table 3: Response for Gravity Beam End Connection Case 2 (k=1.7EI/L, M_{pl}=50%).

	Model 1		Model 2			Model 3		
	As built		5 % Baseshear			10 % Baseshear		
	lognormal mean	deviation	lognormal mean	deviation	change [%]	lognormal mean	deviation	change [%]
Peak Acc Floor 1	7.874	1.665	7.379	1.624	-6.716	7.228	1.643	-8.950
Peak Acc Floor 2	6.837	1.526	5.689	1.653	-20.169	5.582	1.610	-22.475
Peak Acc Floor 3	8.645	1.447	6.094	1.534	-41.866	5.940	1.511	-45.532
Relative Roof Drift	0.194	1.370	0.306	1.380	57.301	0.280	1.387	44.105
Residual Roof Drift	0.015	4.408	0.022	2.934	40.491	0.020	2.273	30.161
Interstorey Drift 3-2	0.069	1.353	0.114	1.289	65.711	0.104	1.310	51.940
Interstorey Drift 2-1	0.066	1.367	0.104	1.378	58.551	0.095	1.384	45.344
Interstorey Drift 1-0	0.067	1.350	0.097	1.342	45.064	0.088	1.359	32.319

Table 4: Response for Gravity Beam End Connection Case 3 (k=2.5EI/L, Mpl=100%)

	Model 1		Model 2			Model 3		
	As built		5 % Baseshear			10 % Baseshear		
	lognormal mean	deviation	lognormal mean	deviation	change [%]	lognormal mean	deviation	change [%]
Peak Acc Floor 1	7.841	1.525	7.315	1.633	-7.192	7.170	1.609	-9.357
Peak Acc Floor 2	6.791	1.434	5.722	1.634	-18.689	5.595	1.620	-21.377
Peak Acc Floor 3	8.738	1.480	6.071	1.543	-43.923	5.953	1.510	-46.776
Relative Roof Drift	0.193	1.367	0.296	1.383	53.576	0.272	1.399	41.343
Residual Roof Drift	0.015	4.832	0.021	3.306	43.479	0.019	3.444	29.323
Interstorey Drift 3-2	0.067	1.362	0.110	1.291	65.111	0.101	1.319	51.888
Interstorey Drift 2-1	0.065	1.362	0.101	1.380	56.059	0.093	1.395	43.738
Interstorey Drift 1-0	0.066	1.352	0.094	1.341	41.970	0.086	1.373	30.037

As seen in Tables 2-4, the total floor accelerations were between 6% and 46% less for the structures with HF2V devices (Models 2 and 3) compared to the as-designed structure (Model 1). The greatest reduction occurred in the upper stories. Accelerations for the 5% and 10% base shear cases (Models 2 and 3) were similar, as were the lognormal standard deviations, indicating no change in potential damage to occupants and contents. However, overall the reductions seen with the devices are significant and reflect significantly reduced damage and thus costs or injury. The similar lognormal deviations indicate that the HF2V design did not change the distribution of the responses, but merely shifted them to lower values. As noted, the reductions in total storey accelerations should significantly increase occupant and contents safety. The bigger damping and the increase in period are reasons for reduced accelerations in the stronger frame.

The median increase in peak storey drift is between 30% and 78%. This result can be attributed to the increased period due to the change made to incorporate the device by weakening selective connections to allow their free movement and use. This increase is greatest in Model 2 (with 5% base shear) for Case 1 where gravity frame effects are ignored because there is no added re-centering stiffness to resist the motion. Note that no damage to structural connections is expected with the HF2V devices in the seismic frame (Models 2-3). Nevertheless, the increase in storey drifts may incur damage to poorly detailed non-structural elements, such as cladding and internal partitions. However, if non-structural elements are designed and detailed to sustain these larger (but not unrealistic) drifts without damage, then no non-structural damage is expected.

Median relative roof drifts are increased by 35% to 94%, which is similar to the increase in drifts. Figure 5 shows the relative roof displacement for the Loma Prieta earthquake record in this medium suite of ground motions and compares the Model 1 Case 2 (M1C2) response to that of Model 2 (5% base shear) Case 2 (M1C2) response for the La01 record. It can be clearly seen that the peak displacement is significantly higher for the 5% base shear design structure.

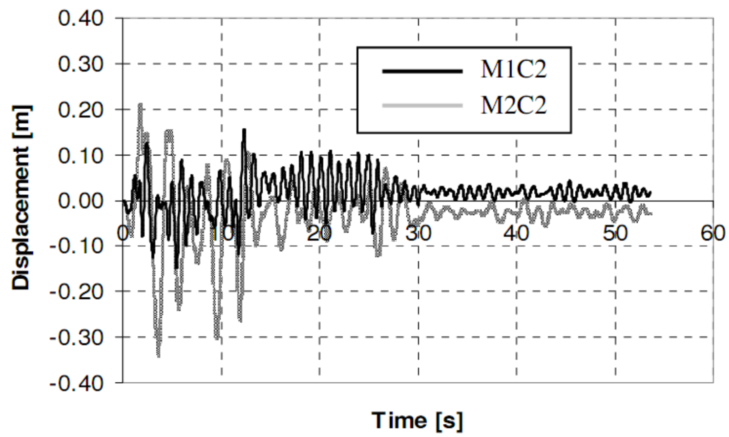

Figure 5: Relative Roof Displacement History for M1C2 and M2C2 with La01 record.

The presence of the HF2V devices (Models 2 and 3) increased the median relative roof residual (or permanent) displacement by almost 100% for the Case 1 frame (with k = 0 at all gravity beam ends). However, for the frames with the gravity beam end connection stiffness and strength considered, the median increase was less than 50%.

Even though the peak displacements of the models containing dissipators, Models 2 and 3, are more than twice that of the as-designed (Model 1, i.e. k=0) structure, the presence of the gravity frame stiffness and strength (i.e. Cases 2 and 3) decreases the residual displacement to about one half that of the as-designed (k=0) structure. The median relative residual roof drifts for the frames with devices when considering the gravity frames was less than 0.22%. This result is close to the allowable construction tolerance of 0.2%. Hence, designing re-centering stiffness into gravity frame connections enables reduced permanent displacement despite damage-free increased transient response.

The distribution of residual drifts is much tighter for the structures considering gravity columns and their re-centering stiffness. For example, the 95th percentile residual drift for Models 1, 2 and 3 were 0.039m * 2.6572 = 0.275m, 0.671m, and 2.243m for the Case 1 (k = 0) frame, but it is 0.291m, 0.189m and 0.103m for the Case 2 frames. The combination of HF2V dissipators with the gravity frame stiffness is therefore very effective and a higher level of confidence can be ascertained to the performance of structures designed with these devices.

Conclusions

This paper has explored the advantages of using HF2V energy dissipators at the beam column joints of steel moment resisting frames by analysing the SAC Los Angeles 3 storey seismic frame subject to the SAC Los Angeles medium suite records. It has incorporated re-centering stiffness both from the gravity columns and from the beam connection to gravity columns together with DAD connections using HF2V devices. The main results of this series of nonlinear finite element analyses can be summarised:

i) Placing HF2V lead dissipators in the seismic frame joints reduces the joint stiffness and increases the fundamental period, resulting in decreased floor accelerations, increased peak displacements and slightly increased residual displacements with respect to those obtained from the as-designed rigid jointed frame. Nevertheless, for frames with well-designed and separated non-structural elements, no damage is expected because all energy dissipation and non-linearity occurs in the damage free HF2V devices. Finally, note that residual displacement could also potentially be ameliorated by (post-event) disconnecting of the HF2V devices to allow the structure to re-center itself if added stiffness braces or gravity frames had not already acted to do so.

ii) Gravity frames provide increased re-centering stiffness. When these were considered in the model, floor accelerations decreased further, and peak displacements decreased but were still greater than that of the as-designed structure. Most significantly, roof residual displacements reduced to approximately 50% of the as-designed structure not considering gravity frame effects.

iii) The combination of HF2V dissipators with gravity frames creates a system with lower floor acceleration and almost no damage despite slightly increased displacements. If drift-sensitive non-structural components are carefully designed, this system appears significantly superior to conventional construction methods, particularly with respect to the resulting economic, financial and business impacts.

Acknowledgements

This work was partially supported by the NZ Heavy Engineering Research Association and John Jones Steel Limited. The travel support of the New Zealand Earthquake Commission is gratefully acknowledged.

References

Bradley, B. A., Dhakal, R. P., Mander, J. B., and Li, L. (2008). "Experimental multi-level seismic performance assessment of 3D RC frame designed for damage avoidance." Earthquake Engineering & Structural Dynamics, 37(1), 1-20.

Building Seismic Saftey Council (BSSC). (2003). "National Earthquake Hazard Reduction Program (NEHRP) Recommended Provisions for Seismic Regulations for New Buildings and Other Structures - FEMA 450." prepared by the Building Seismic Safety Council (BSSC) for the Federal Emergency Management Agency (FEMA), Washington DC.

Cousins, W. J., and Porritt, T. E. (1993). "Improvements to lead-extrusion damper technology." Bulletin of the New Zealand National Society for Earthquake Engineering, 26(3), 342-348.

Kawashima, K., MacRae, G. A., Hoshikuma, J.-i., and Nagaya, K. (1998). "Residual displacement response spectrum." Journal of Structural Engineering, 124(5), 523-530.

Krawinkler, H., and Gupta, A. (1998). "Story Drift Demands for Steel Moment Frame Structures in Different Seismic Regions." 6th US National Conference on Earthquake Engineering, Seattle, Washington, USA.
Li, L. (2006). "Further Experiments on Damage Avoidance design of Beam-to-column joints," Master of Engineering Thesis, University of Canterbury, Christchurch, New Zealand.
Limpert, E., Stahel, W. A., and Abbt, M. (2001). "Log-normal distributions across the sciences: keys and clues." Bioscience 2001; vol 51(5), pp. 341-352.
Liu, J., and Astaneh-Asl, A. (2000). "Cyclic testing of simple connections including effects of slab." Journal of Structural Engineering, 126(1), 32-39.
Liu, J., and Astaneh-Asl, A. (2004). "Moment-rotation parameters for composite shear tab connections." Journal of Structural Engineering, 130(9), 1371-1380.
MacRae, G. A., and Kawashima, K. (1997). "Post-earthquake residual displacements of bilinear oscillators." Earthquake Engineering & Structural Dynamics, 26(7), 701-716.
MacRae, G. A., Kimura, Y., and Roeder, C. (2004). "Effect of column stiffness on braced frame seismic behavior." Journal of Structural Engineering, 130(3), 381-391.
Mander, T., Rodgers, G., Chase, J., Mander, J., MacRae, G., and Dhakal, R. (2009). "A Damage Avoidance Design Steel Beam-Column Moment Connection Using High-Force-To-Volume Dissipators." Journal of Structural Engineering (JSE), in-press.
Robinson, W. H. (1995). "Recent research and applications of seismic isolation in New Zealand." Bulletin of the New Zealand National Society for Earthquake Engineering, 28(4), 253-264.
Robinson, W. H., and Greenbank, L. R. (1976). "Extrusion Energy Absorber Suitable For The Protection Of Structures During An Earthquake." Earthquake Engineering & Structural Dynamics, 4(3), 251-259.
Rodgers, G., Solberg, K., Chase, J., Mander, J., Bradley, B., Dhakal, R., and Li, L. (2008a). "Performance Of A Damage-Protected Beam-Column Subassembly Utilizing External HF2V Energy Dissipation Devices." Earthquake Engineering & Structural Dynamics (EESD), 37(3), 1549-1564.
Rodgers, G. W., Chase, J. G., Mander, J. B., Leach, N. C., and Denmead, C. S. (2007). "Experimental development, tradeoff analysis and design implementation of high force-to-volume damping technology." Bulletin of the New Zealand Society for Earthquake Engineering, 40(2), 35-48.
Rodgers, G. W., Mander, J. B., Chase, J. G., Dhakal, R. P., Leach, N. C., and Denmead, C. S. (2008b). "Spectral analysis and design approach for high force-to-volume extrusion damper-based structural energy dissipation." Earthquake Engineering & Structural Dynamics (EESD), 37(2), 207-223.
Rodgers, G. W., Solberg, K. M., Mander, J. B., Chase, J. G., Bradley, B. A., Dhakal, R. P., and Li, L. (2008c). "Performance Of A Damage-Protected Beam-Column Subassembly Utilizing External HF2V Energy Dissipation Devices." Earthquake Engineering & Structural Dynamics, 37(13), 1549-1564.
SAC. (1999). "The SAC Steel Project: Seismic Design Criteria for Steel Moment Frame Construction."
Solberg, K. (2007). "Experimental and Financial Investigations into the further development of Damage Avoidance Design," University of Canterbury, Christchurch, New Zealand.
Sommerville, P., Smith, N., Punyamurthula, S., and Sun, J. (1997). "Development of Ground Motion Time Histories For Phase II Of The FEMA/SAC Steel Project, SAC Background Document Report SAC/BD-97/04."